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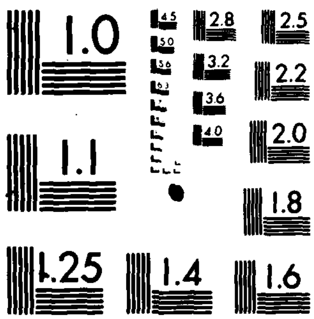

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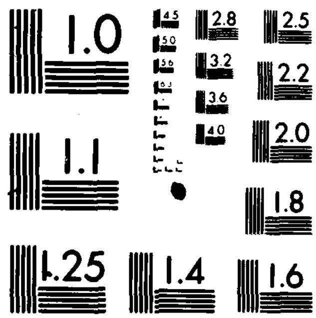

MICROCOPY RESOLUTION TEST CHART

NATIONAL BUREAU OF STANDARDS 1963-A

MISCELLANEOUS PAPER SL-82-1

DURABILITY OF REINFORCED CONCRETE BEAMS EXPOSED TO MARINE ENVIRONMENT

by

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April 1982
Final Report

Approved For Public Release; Distribution Unlimited

Prepared for Office, Chief of Engineers, U. S. Army
Washington, D. C. 20314

Under Civil Works Research Work Unit 010401/31276

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER: Miscellaneous Paper SL-82-1 2. GOVT ACCESSION NO.: AD-A119284	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle): DURABILITY OF REINFORCED CONCRETE BEAMS EXPOSED TO MARINE ENVIRONMENT	5. TYPE OF REPORT & PERIOD COVERED: Final report 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s): Edward F. O'Neil	8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS: U. S. Army Engineer Waterways Experiment Station, Structures Laboratory, P. O. Box 631, Vicksburg, Miss. 39180	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS: Civil Works Research Work Unit 010401/31276
11. CONTROLLING OFFICE NAME AND ADDRESS: Office, Chief of Engineers, U. S. Army, Washington, D. C. 20314	12. REPORT DATE: April 1982 13. NUMBER OF PAGES: 18
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)	15. SECURITY CLASS. (of this report): Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Available from National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22151.

This is CTIAC Report No. 49.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Concrete--Deterioration
Concrete--Testing
Concrete beams
Reinforced concrete beams
Weathering

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A study was begun in 1950 to determine the effects of severe natural weathering on stressed, reinforced concrete beams of various compositions and degrees of stress. The objectives of the study were to obtain information on the long-term weathering of air-entrained and nonair-entrained concrete beams containing steel of different compositions and types of deformation and having different levels of stress in the steel that would cause varying degrees of cracking of the concrete.

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Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. ABSTRACT (Continued)

The beams were fabricated, cured, and loaded at the U. S. Army Engineer Waterways Experiment Station (WES) in 1951, then shipped to Eastport, Maine, and placed on the beach at the natural weathering exposure station on the south side of Treat Island, Cobscock Bay, Eastport, and Lubec. The beams were subjected to twice daily tidal cycles exposing them to wetting under considerable head and drying to surface dry conditions. In addition, during the winter months, the beams were subjected to cycles of freezing and thawing with each tide when the air temperature was at or below -2.2°C. The beams were inspected annually during the exposure period and evaluated by a team of inspectors to determine the degree of deterioration. Maximum crack widths were also measured annually beginning in 1956 and continuing until 1975, when the exposure period was concluded after 25 years of weathering.

At the end of the exposure period, 13 of the 82 beams still remained at the testing site. (By 1956, 60 of the beams, all fabricated from nonair-entraining concrete, had been destroyed by freezing and thawing.) Of the 13 beams remaining in 1975, 11 were returned to the laboratory for laboratory testing.

The results of the exposure study and the laboratory investigation indicated, among other findings, that stressing the steel to various levels of stress over the exposure period did not adversely reduce the moment carrying capacity of the beams even though rusting and spalling had occurred; corrosion to the steel could not be found at any flexural crack smaller than 0.40 mm; and sustained levels of stress to the reinforcing did not reduce the tensile properties of the steel to below acceptable standards.

PREFACE

This report is essentially a paper prepared for presentation at the International Conference on the Performance of Concrete in the Marine Environment. This conference, jointly sponsored by the Canada Centre for Mineral and Energy Technology (CANMET) of Energy, Mines, and Resources Canada; the American Concrete Institute (ACI); the University of New Brunswick; the U. S. Army Corps of Engineers; the Structural Division of the Canada Society for Civil Engineering; and the Eastern Ontario, Quebec Region, and Atlantic Chapters of ACI, was held 17-22 August 1980 at St. Andrews, New Brunswick, Canada.

The research that produced the results presented in this paper was conducted for the Office, Chief of Engineers, U. S. Army, as part of Civil Works Research Work Unit 010401/31276, "Reinforced Concrete Beams for Tensile Crack Exposure Tests." The research was done in the Concrete Technology Division (CTD) of the Structures Laboratory (SL), U. S. Army Engineer Waterways Experiment Station (WES). Original approval for the investigation was given in the 2nd indorsement, dated 17 January 1951, to a basic letter, dated 7 December 1950.

Funds for publication of the report were provided from those made available for operation of the Concrete Technology Information Analysis Center (CTIAC). This is CTIAC Report No. 49. The paper was prepared by Mr. Edward F. O'Neil under the general supervision of Messrs. Bryant Mather, Chief, SL; John M. Scanlon, Chief, CTD; and James E. McDonald, Chief, Evaluating and Monitoring Group.

Commander and Director of WES during publication of this report was COL T. C. Creel, CE. Technical Director was Mr. F. R. Brown.

A

CONTENTS

DURABILITY OF REINFORCED CONCRETE BEAMS EXPOSED TO MARINE ENVIRONMENT

INTRODUCTION

In the early 1950's one of the objectives of the Corps of Engineers exposure studies at the Treat Island Exposure Station was to determine the effects of sustained stress on the durability of reinforced concrete beams exposed to severe natural weathering. The thinking behind the studies was to obtain information on the long-term weathering of air-entrained and nonair-entrained beams containing steels of different composition and deformations, and having different stress levels that caused varying degrees of cracking of the concrete. A detailed description of the tests and results presented here may be found in the final report of the Tensile Crack Exposure Series (1) of the WES.

Description of the Beams

A series of 82 beams was fabricated. These beams contained a number of variables that might affect the durability of the concrete and the steel used as reinforcement. Eighteen of the 82 beams were made with air-entrained concrete; the rest were made from a similar mixture without air entrainment to evaluate the desirability of air entrainment in severe environments. Thirty-nine percent of the beams were cast with 50-mm cover over the reinforcement while the remaining beams had 19-mm cover to evaluate depth of cover over the reinforcement. The reinforcing steel in the 82 beams conformed to ASTM Designation A 16-50T for "Rail-Steel Bars for Concrete Reinforcement," or to Designation A 15-50T for "Billet-Steel Bars for Concrete Reinforcement," intermediate grade. The billet-steel bars conformed to ASTM Designation A 305-50T for "Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement." Some of the rail-steel bars had deformations conforming to ASTM Designation A 305-50T, and the others had old-style deformations that did not meet the requirements of A 305. The different types of steel and types of bar deformations were used to evaluate the resistance to deterioration of rail steel versus billet steel and to see if old-style deformations or ASTM Designation A 305 type deformations provided better pullout resistance. Thirty-three percent of the beams were cast in an inverted position with their steel reinforcement in the top of the form, and the rest were cast in the upright position with the steel in the bottom of the forms. This was done to determine if the steel cast in the bottom of the beam would weather better than steel cast in the top of the beam, due to greater amounts of cement at the top of the beam from segregation during casting.

Stress Levels

All the beams, with the exception of the control beams, were loaded to put the tensile steel under stress. Beams requiring the same amount of load were yoked together in third point flexural loading. The beams were loaded such that the various levels of stress in the steel were 0, 138, 207, 276, and 345 MPa. Table 1 is a brief synopsis of the beams described in this paper.

Exposure Conditions

The beams were fabricated, cured, and loaded at the WES in 1951, then shipped to Maine and placed on the beach at the natural weathering exposure station on the south side of Treat Island, which is located in Cobscock Bay between Eastport and Lubec. There, they were subjected to twice daily tidal cycles (the average tidal range is 5.5 m, occasionally reaching maximums of 9.15 m) exposing them to wetting under considerable head and drying to surface dry conditions. In addition, during the winter months, the beams were subjected to cycles of freezing and thawing with each tide when the air temperature was at or below $-2.2^{o}C$ (freezing point of seawater). One cycle of freezing and thawing was completed each time the temperature of the center of the beam passed below $-2.2^{o}C$ and then above $-2.2^{o}C$. The number of freeze/thaw cycles has ranged from 85 to 242 per year with an average of 135 cycles per year over the exposure period from September 1951 to December 1975.

The loading of the beams caused them to be cracked in their tensile zones, as shown in Figure 1, of two beams in their loaded, yoked condition. This cracking was a potential source for the ingress of salt water into the beam and a condition for accelerated corrosion to the steel.

Periodic Inspections

Inspection of the beams was conducted by the resident inspector weekly. At these inspections adjustments were made to the loading yokes to maintain the gap openings of the spacer gages. In addition to this weekly maintenance, the specimens were inspected annually by a team of observers to evaluate the deterioration of the beams. The inspectors rated the condition of the beams with respect to visual deterioration such as spalls and rust stains, exposure of the reinforcing steel, and loss of load carrying capacity.

By the end of January 1956 all of the nonair-entrained beams had deteriorated due to freezing and thawing to the point of failure to carry load. The testing and observation to these beams was discontinued at this point in the exposure period, leaving the 18 air-entrained beams to continue the exposure.

Adaptation of Study Parameters

Sixty of the 82 beams in this series were nonair-entrained beams; and since all of these beams deteriorated to a point of total failure within 5 years after initiation of testing, some of the variables were eliminated because of failure of the specimens and lack of parameters for comparison. The nonair-entrained beams contained all the billet-steel reinforcing bars, all the bars that were protected by 50 mm of cover, and all the reinforcing bars that had old-style deformations; consequently, all these variables were lost when the nonair-entrained beams failed. The variables remaining subsequent to January 1956 which could be compared were the degree of stress in the reinforcing steel, the amount of corrosion to the bar, and the effect of casting the reinforcement in the top or bottom of the beam.

An additional parameter was added to the evaluation of the exposure tests. This was the relation of crack width to the amount of corrosion on the steel. Also, the loss of cross sectional area of steel and chloride content determination tests were added to the testing program.

LABORATORY TESTS AT END OF EXPOSURE PERIOD

At the conclusion of the exposure period in December 1975, 13 of the 18 air-entrained beams were still in testable condition. Eleven of these 13 beams were returned to the WES for structural testing and autopsy to evaluate the condition of the beams after 25 years of exposure. A typical beam as it was received in the laboratory is shown in Figure 2.

Flexural Failure Tests

Seven of the 11 beams were tested to failure in third-point flexural loading. The beams were marked at supports and third points of the 216-cm test span, centered in the testing frame, and checked for longitudinal alignment and levelness. Two beams from the 138-MPa range (No. 2 and 4), three beams from the 207-MPa range (No. 5, 7, and 8), and one beam from the 276-MPa range (No. 12), as well as beam No. 18, which was exposed without being stressed, were flexurally loaded to failure at a constant loading rate of 8.9 kN per minute. The midspan deflection was measured by dial gage with least reading of 0.02 mm at every 8.9-kN increment of load until failure. At failure the ultimate load and deflection were recorded and these data are given in Table 2.

Autopsy Examination

The beams were examined to determine the amount of corrosion to the reinforcing steel. When they were received in the laboratory, sketches were made to show the extent of spalling of the reinforcement cover. The length of the concrete spall, the length of exposed steel, and the distance from the end of the beam were recorded. A sketch of the flexural cracks and their dimensions and locations was recorded. The cover over the steel was then broken away and the steel removed from the beam. Sketches of the corrosion to the steel were recorded to be matched to the crack locations of the concrete (Figures 3 through 8).

Chloride Content

Samples of the concrete were taken across the cross section of the beams at midspan to determine the distribution of chlorides in the concrete at the level of the steel. The analysis was made by a silver nitrate titration test described by Berman (2). The results of the tests are discussed later in this report.

Ultimate Tensile Strength of the Steel

Selected portions of the reinforcement from the beams were tension tested to ultimate load to determine the ultimate strength and the stress-strain properties of the bars. Three-foot-long sections of reinforcing bars were selected from the least corroded area of each bar to eliminate the factor of reduction in strength due to corrosion from the comparison of tensile properties of the steel with stress level. One 19-mm diameter bar each from the 0-, 138- and 207-MPa stress level and one 16-mm diameter bar from the 276-MPa stress level were chosen to provide specimens that were in approximately the same condition and nearly the same diameter. The results of the structural testing were compared with ASTM Designation A 16-57T for rail-steel bars used as concrete reinforcement. Structural testing results are given in Table 2.

DISCUSSION OF TEST RESULTS

Flexural Tests

The beams that were tested in flexure to determine their ultimate load were beams representing stress levels of 138,* 138B, 207T, 207B, 276B, and unstressed. Four of these beams arrived at the WES broken at centerspan from improper handling during transfer to the laboratory. They were tested in third-point flexure in the same manner as the unbroken beams.

*T means beam cast inverted; B means beam cast upright.

Although they were cracked all the way through their section, the method of loading placed part of the broken section back in compression and all the tensile stress on the reinforcement. This was similar to the stress conditions of the unbroken beams since they also had compression on the upper section and all tensile stress on the reinforcement (due to the deliberately cracked section of the test program). Whether or not the beams were broken when tested to failure, ultimate load ranged from 127 kN to 177kN (Table 2).

All the beams failed in diagonal tension, with six of the failures initiated by pullout of the reinforcement from the concrete at the end experiencing the diagonal tension failure.

The ratio comparison given in the last column of Table 2 is the actual ultimate moment developed in the beams during flexural loading compared to the design ultimate moment calculated using concrete and steel data from the beams. These ratios are significant where the failure loads are not because of the differences in beam cross section and amount of reinforcing steel at the different stress levels. The moment ratio developed at failure ranges from 0.839 in the control beam to 1.753 at the 276-MPa stress level. The design ultimate moment was that calculated by ultimate strength requirements without a safety factor applied. The yield strength of the steel used in the calculations was 345 MPa, which was the minimum specified yield strength for rail-steel reinforcement at the time of casting. This assumption is fortified by the yield strength tests conducted during the laboratory testing period. The data in Table 2 show that with the exception of the control beam, all actual ultimate moments were greater than the design ultimate moments. The ratio of 1.753 of the beam at the 276-MPa stress level is of doubt because its deflection at midspan is twice that of any other beam, and the ultimate load recorded was probably a load recorded after failure. No definite relationship can be drawn from the data with respect to ultimate moment and stress level. However, it can be stated that the ultimate moments were not reduced below design levels over the 25-year period due to level of stress, corrosion of the reinforcement, or loss of bond length from spalling except in the control beam.

Casting Position of the Steel

Observations of the imprint of the reinforcing bar in the paste surrounding the steel showed that the paste above the bar was more dense, contained fewer air voids, and was harder to scratch. The areas of paste below the bars were chalky in texture, contained more air voids, and were less dense. Another observation that was made was that regardless of whether the reinforcement was cast in the top or the bottom of the beam, the hardened paste did not segregate from the bar enough to destroy the pattern of the deformations in the paste. If the paste had segregated from the bar to a point where it no longer interlocked with the bar deformations, then in addition to lost bond the benefits of the deformations in transferring stresses from the concrete to the steel would also be lost. This phenomenon did not occur on any samples of concrete examined in this investigation.

Corrosion to the Reinforcement

As a general remark, the steel beneath the 19-mm cover was not heavily rusted. There were areas that did receive corrosion where the concrete cover was still intact, such as the tips of the reinforcing bars, but on the whole, areas where the bars were not directly exposed to oxygen and seawater remained only lightly rusted. Where the concrete cover had spalled, the steel was heavily rusted from direct exposure to the harsh environment.

Figures 3 through 8 are graphic representations of the beams in the "as received" condition. Each figure shows both faces of the beam and all areas of the reinforcement that were exposed. They also show the longitudinal and flexural cracks, spalled areas, and corrosion to the reinforcement. The sketch between the faces of the beams shows the condition of the reinforcement after it was removed from the concrete. The darkened areas on the bars represent the corroded areas. The figures show that the number and depth of flexural cracks increased with the increase in steel stress. They also show that the corrosion on the bars does not align with the flexural cracks in the concrete at lower steel stress levels. The corrosion on the bars that were stressed in the 138-MPa stress range was, in general, at different locations than the flexural cracks. The corrosion at spalled areas was confined to the area of the steel directly exposed to water and oxygen; however, where there were flexural cracks and the concrete was not spalled, the bars beneath remained free from corrosion. In the higher stress ranges, 207 and 276 MPa, the corroded areas of the steel more generally matched the location of the flexural cracks (Figures 6 and 7).

Stress Level Versus Corrosion

The data gathered by the teams of observers on the measurements of crack widths over the period 1957 through 1975 indicate an increase in crack width with respect to both length of time under load and level of stress applied to the reinforcement. At the most recent annual inspection where all stress levels could be compared on an equal basis, the beams of the 345-MPa stress level had the largest flexural cracks, and the beams at the 138-MPa stress level had the smallest. The data also indicate gradual crack opening as time progressed ranging from a maximum of 0.4 mm (in the 345-MPa stressed beams) at the start of crack width measurement in 1957 to a maximum of 1.4 mm (345 MPa) at conclusion of testing in 1975. As mentioned earlier, no corrosion to the steel could be matched with the flexural cracks in the beams at the 138-MPa stress range, and at the higher ranges corrosion began to show up at flexural cracks. At the 138-MPa stress level the smallest value of crack width (as measured in 1971) was 0.4 mm. This indicates that corrosion to the steel did not occur at cracks of less than 0.4 mm in width.

It was observed that there was more corrosion on the bars at the higher stress levels. This was due to the larger crack widths and the greater exposure to oxygen and seawater. Also, the results of the chloride content tests revealed concentrations of chlorides ranging from 0.12 to 0.70 percent by weight of concrete sample, a chloride concentration high enough to severely reduce the passivating coating on the reinforcing steel. However, no relationship between stress level and loss of cross sectional area due to corrosion could be established since the control beam which was under no steel stress had an area of cross section reduced 33 percent, which was the second largest reduction observed. With respect to stress level and corrosion to the steel, it appears that they are not related except through the fact that the higher stress levels produced larger crack widths which allowed greater influx of the corrosive environment.

Stress Level Versus Tensile Properties of the Steel

The stress-strain properties of selected pieces of steel were taken to determine if the stress level during exposure had any effect on the properties of the steel. The samples were taken from the reinforcement in areas of the bar where there was minimum corrosion. All bars tested exhibited greater than minimum specifications of ultimate tensile strength and yield point according to ASTM Designation A 16-57T. The ultimate strength was calculated as the ultimate tensile load divided by the cross sectional area of the bar, and the yield point was taken as that point in the stress-strain curve where the curve deviated from linearity of the elastic range. The data showed a decrease in the yield point of the steel with an increase in load; however, this is not considered sufficient to indicate a reduction in strength with increasing stress level during exposure since this trend is not reflected in the ultimate strength data.

The steel tensile properties did satisfy the minimum ASTM standards for rail-steel after 25 years of severe exposure. It should be mentioned that the steel specimens were taken from areas of the bar where there was only mild surface rust and that the data presented does not represent areas of the steel that were directly exposed to the environment.

CONCLUSIONS

Data extracted from the 25 years of exposure study of the reinforced concrete beams subjected to various states of steel stress have revealed the following conclusions:

The use of an air-entraining admixture greatly improved the durability of the concrete.

The quality of the paste around the reinforcement was better on the top side of the reinforcing bar than that on the bottom, but there was no evidence that casting the reinforcement in the bottom of the form gave any better pullout resistance over casting the beam inverted with the reinforcement in the top of the form.

Even though all the beams exhibited some spalling, ultimate load tests revealed that only one of the beams failed at an ultimate moment lower than design ultimate moment, indicating that the years of exposure did not reduce ultimate load carrying capacity below design levels.

There was no relationship between level of stress in the steel during exposure and ultimate load experienced in the beams at testing to failure.

The steel was found to be heavily corroded at spalled areas and generally free from corrosion beneath the 19-mm cover. Since corrosion could not be found at any cracks in the beams stressed at the 138-MPa level, it is concluded that crack widths greater than 0.4 mm were necessary to produce corrosion at flexural cracks.

No definite relationship between the stress in the steel and the amount of corrosion could be made that would indicate that steel at higher stress levels should corrode more or less than steel at lower stress levels; but it was generally found that the steel in beams subjected to the higher stress levels was more corroded because the higher loads induced wider flexural cracks for the ingress of more water and oxygen.

The level of stress to which the steel was exposed did not adversely affect the ultimate tensile properties of the steel when tested to failure.

No conclusions could be formed with respect to the differences in amounts of corrosion on rail steel versus billet steel or old-style reinforcement deformations versus A 305 deformations; nor could any conclusion be made with respect to depth of cover over the reinforcement (50 mm versus 19 mm) because comparative specimens in all of these variables were destroyed early in the exposure period due to lack of air entrainment in the concrete.

REFERENCES

1. O'Neil, E. F., "Tensile Crack Exposure Tests; Laboratory Evaluation of Series 'A' Beams with Results from 1951 to 1975," TM 6-412, Report 3, US Army Engineer Waterways Experiment Station, CE, Vicksburg, MS, 1980.

2. Berman, H. A., "Determination of Chloride in Hardened Portland Cement Paste, Mortar, and Concrete," Journal of Materials, American Society for Testing and Materials, Vol 7, No. 3, Sep 1972, pp 330-335.

Table 1

Synopsis of Variables in Beams at the Start of Exposure Testing

Beam No.	Concrete	Deformation*	Nom Stress, MPa	Cover, mm	Steel Position
1	Air	A	138	19	Top
2	Air	A	138	19	Top
3	Air	A	138	19	Bottom
4	Air	A	138	19	Bottom
5	Air	B	207	19	Top
6	Air	B	207	19	Top
7	Air	B	207	19	Bottom
8	Air	B	207	19	Bottom
9	Air	A	276	19	Top
10	Air	A	276	19	Top
11	Air	A	276	19	Bottom
12	Air	A	276	19	Bottom
13	Air	B	345	19	Top
14	Air	B	345	19	Top
15	Air	B	345	19	Bottom
16	Air	B	345	19	Bottom
17	Air	B	Control	19	Bottom
18	Air	B	Control	19	Bottom
19	Nonair	B	138	19	Top
20	Nonair	B	138	19	Bottom
21	Nonair	B	138	19	Bottom
22	Nonair	B	138	19	Bottom
23	Nonair	Old-S	138	19	Bottom
24	Nonair	Old-S	138	19	Bottom
25	Nonair	A	207	19	Top
26	Nonair	A	207	19	Bottom
27	Nonair	A	207	19	Bottom
28	Nonair	A	207	19	Bottom
29	Nonair	Old-S	207	19	Bottom
30	Nonair	Old-S	207	19	Bottom
31	Nonair	B	276	19	Top
32	Nonair	B	276	19	Top
33	Nonair	B	276	19	Top
34	Nonair	B	276	19	Bottom
35	Nonair	B	276	19	Bottom
36	Nonair	B	276	19	Bottom
37	Nonair	A	345	19	Top
38	Nonair	A	345	19	Top
39	Nonair	A	345	19	Top
40	Nonair	A	345	19	Bottom

(Continued)

* "A" bars have deformations higher and more closely spaced than ASTM Spec A 305; "B" bars meet ASTM Spec A 305; Old-Style bars do not meet A 305.

Table 1 (Continued)

Beam No.	Concrete	Deformation*	Nom Stress, MPa	Cover, mm	Steel Position
41	Nonair	A	345	19	Bottom
42	Nonair	A	345	19	Bottom
43	Nonair	B	--	19	Top
44	Nonair	B	--	19	Bottom
45	Nonair	B	--	19	Bottom
46	Nonair	Old-S	--	19	Bottom
47	Nonair	A	138	50	Top
48	Nonair	A	138	50	Bottom
49	Nonair	A	138	50	Bottom
50	Nonair	A	138	50	Bottom
51	Nonair	Old-S	138	50	Bottom
52	Nonair	Old-S	138	50	Bottom
53	Nonair	B	207	50	Top
54	Nonair	B	207	50	Bottom
55	Nonair	B	207	50	Bottom
56	Nonair	B	207	50	Bottom
57	Nonair	Old-S	207	50	Bottom
58	Nonair	Old-S	207	50	Bottom
59	Nonair	A	276	50	Top
60	Nonair	A	276	50	Top
61	Nonair	A	276	50	Top
62	Nonair	A	276	50	Bottom
63	Nonair	A	276	50	Bottom
64	Nonair	A	276	50	Bottom
65	Nonair	B	345	50	Top
66	Nonair	B	345	50	Top
67	Nonair	B	345	50	Top
68	Nonair	B	345	50	Bottom
69	Nonair	B	345	50	Bottom
70	Nonair	B	345	50	Bottom
71	Nonair	A	--	50	Top
72	Nonair	A	--	50	Top
73	Nonair	A	--	50	Bottom
74	Nonair	Old-S	--	50	Bottom
75	Nonair	A	138	19	Bottom
76	Air	A	138	19	Bottom
77	Nonair	A	138	50	Bottom
78	Air	A	138	50	Bottom
79	Nonair	A	207	19	Bottom
80	Air	A	207	19	Bottom
81	Nonair	A	207	50	Bottom
82	Air	A	207	50	Bottom

* "A" bars have deformations higher and more closely spaced than ASTM Spec A 305; "B" bars meet ASTM Spec A 305; Old-style bars do not meet A 305.

Table 2

Ultimate Load Properties of Beams Tested in Flexure

Beam No.	Stress Level, MPa	Stress Position	Failure Load, kN	Midspan Deflection, mm	Moment Developed at Ultimate Load, N·m	Design Ultimate Moment N·m	Ratio $\frac{M'_u \text{ Actual}}{M'_u \text{ Design}}$
2	138	Top	177	11.2	63,696	58,748	1.084
4	138	Bottom	169	13.1	60,795	58,748	1.035
5	207	Top	154	20.1	55,521	45,664	1.216
7	207	Bottom	127	20.3	45,759	45,664	1.002
8	207	Bottom	168	12.4	60,469	45,664	1.324
12	276	Bottom	150	40.8	53,989	30,791	1.753
18	Unstressed	Bottom	137	6.1	49,270	58,748	0.839

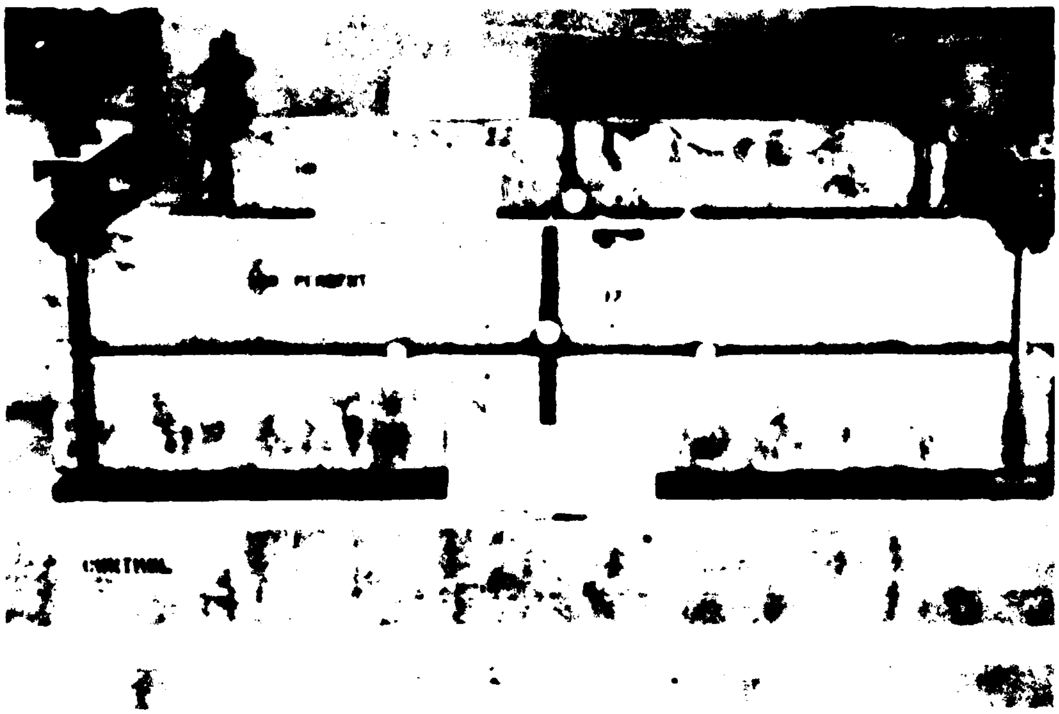

Figure 1. Typical beams as prepared for exposure

Figure 2. As received condition of a typical beam

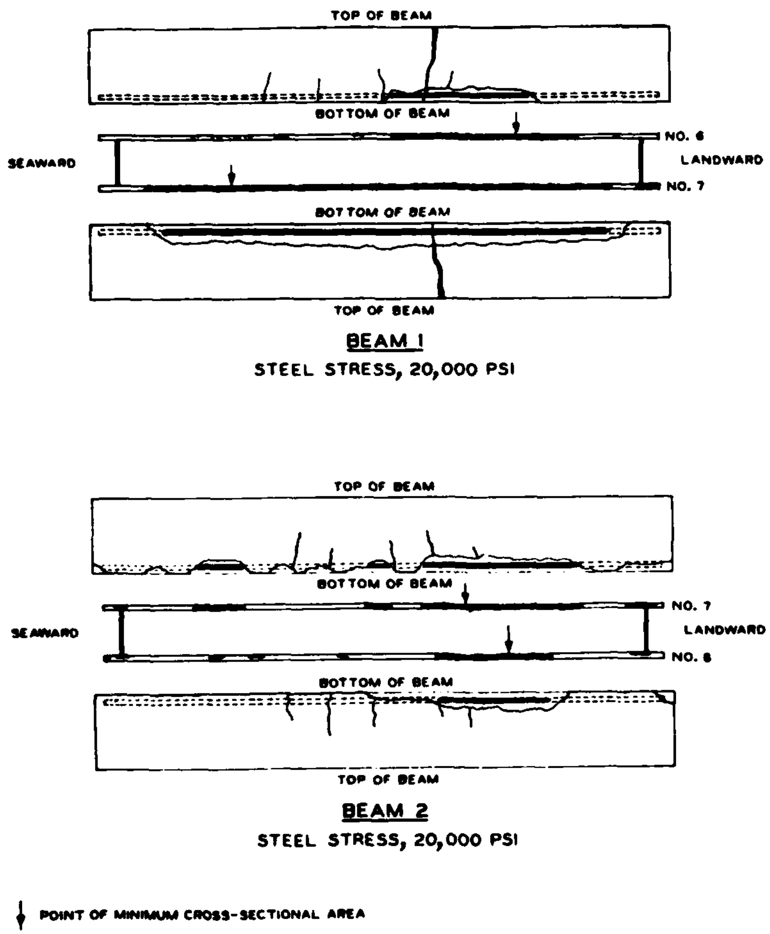

Figure 3. Graphic representation of cracking and corrosion, beams 1 and 2

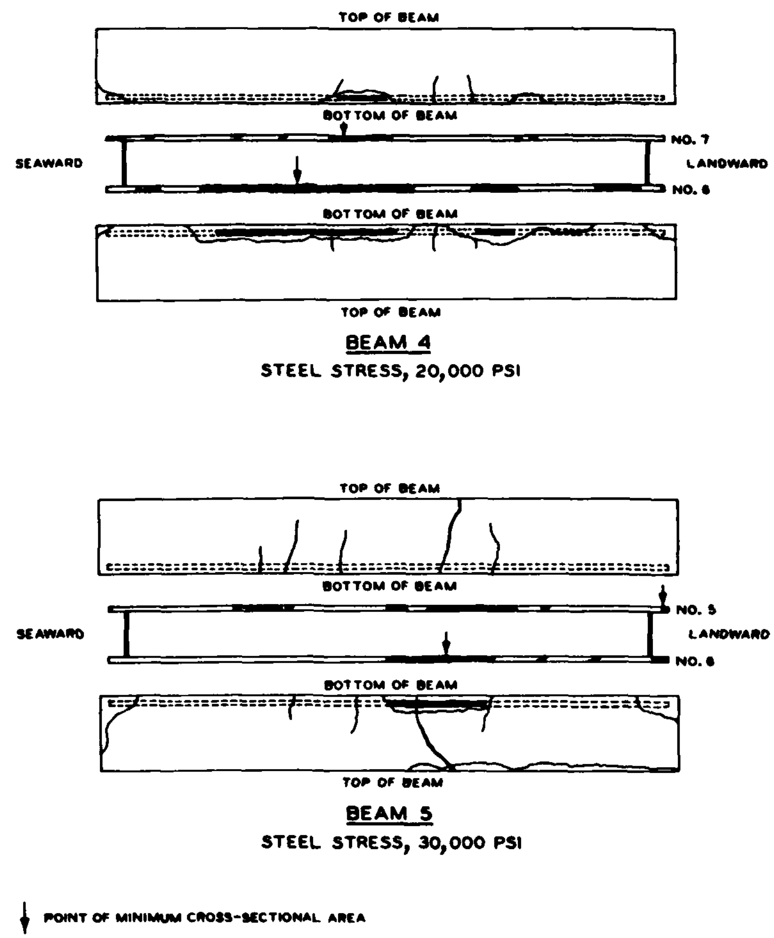

Figure 4. Graphic representation of cracking and corrosion, beams 4 and 5

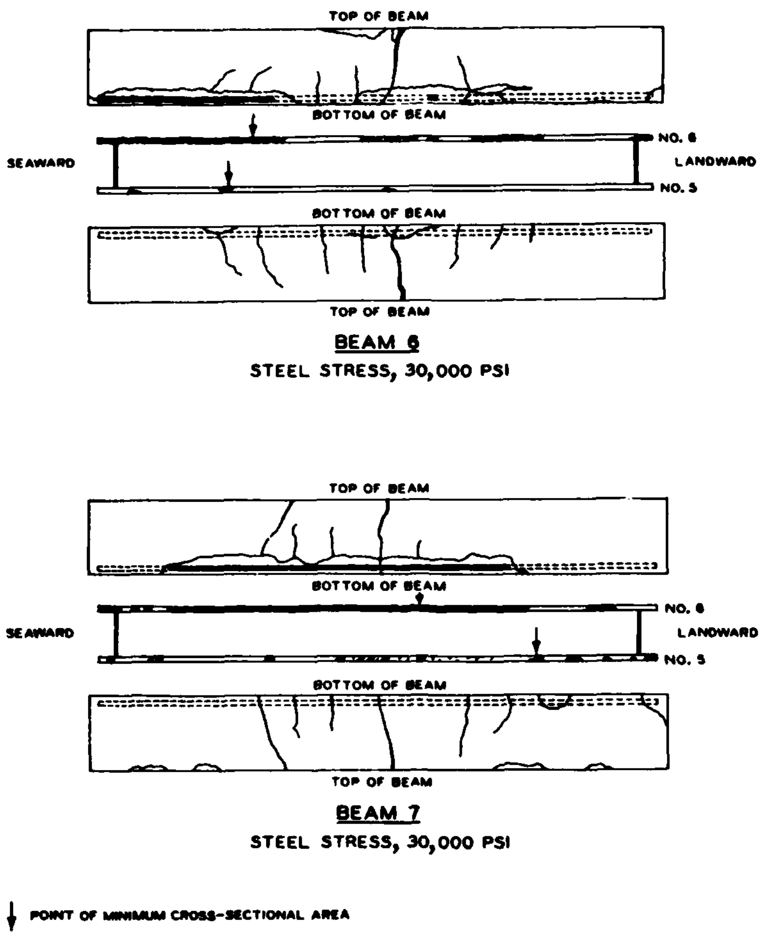

Figure 5. Graphic representation of cracking and corrosion, beams 6 and 7

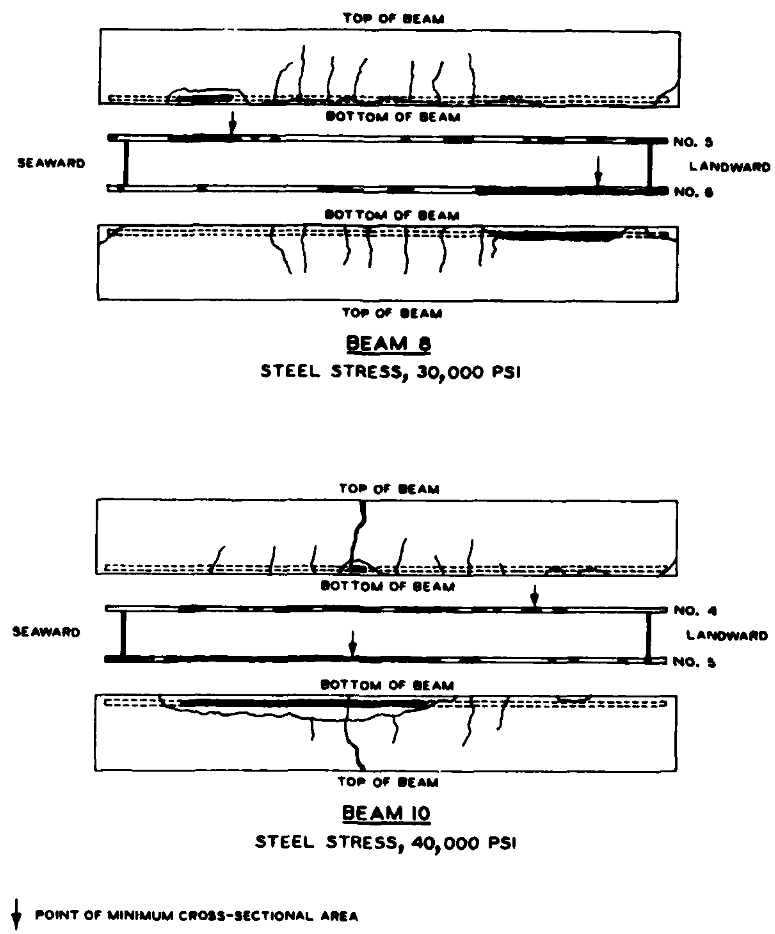

Figure 6. Graphic representation of cracking and corrosion, beams 8 and 10

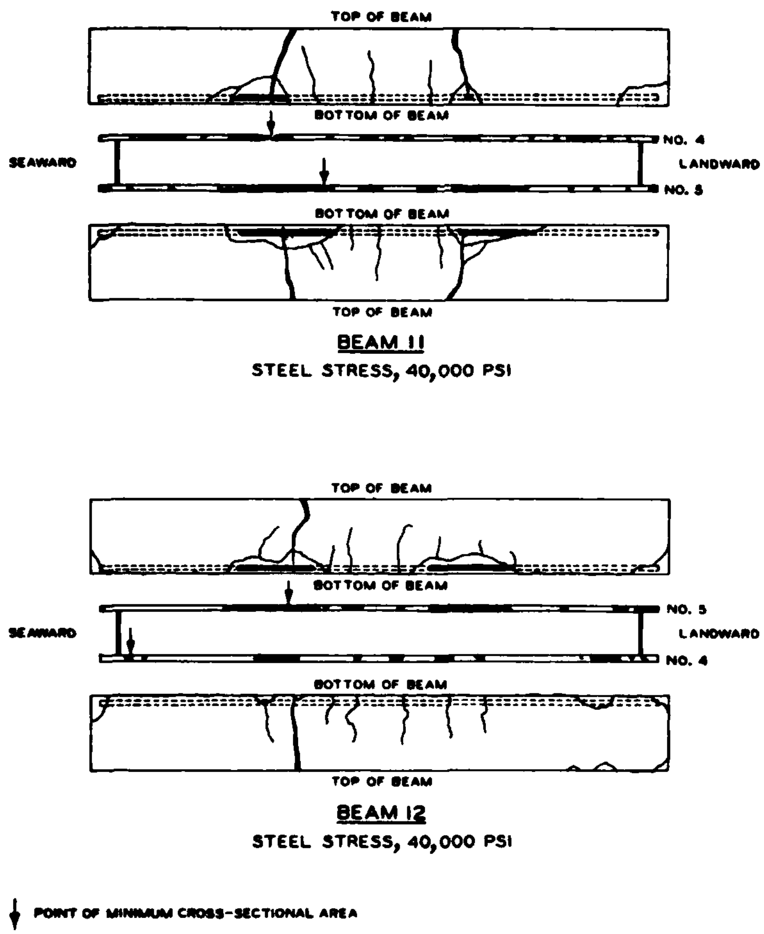

Figure 7. Graphic representation of cracking and corrosion, beams 11 and 12

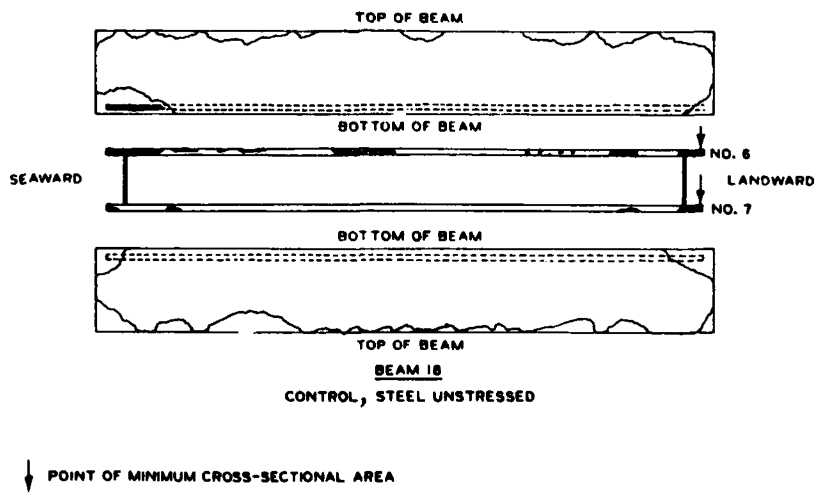

Figure 8. Graphic representation of cracking and corrosion, beam 18

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

O'Neil, Edward F.
  Durability of reinforced concrete beams exposed to marine environment / by Edward F. O'Neil (Structures Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, 1982.
  11, [7] p. : ill. ; 27 cm. -- (Miscellaneous paper ; SL-82-1)
  Cover title.
  "April 1982."
  Final report.
  "Prepared for Office, Chief of Engineers, U.S. Army under Civil Works Research Work Unit 010401/31276."
  Bibliography: p. 11.

  1. Concrete--Deterioration. 2. Concrete--Testing. 3. Concrete beams. 4. Reinforced concrete beams. 5. Weathering. I. United States. Army. Corps of Engineers. Office of the Chief of Engineers. II. U.S. Army

O/Neil, Edward F.
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Engineer Waterways Experiment Station. Structures Laboratory. III. Title IV. Series: Miscellaneous paper (U.S. Army Engineer Waterways Experiment Station) ; SL-82-1.
TA7.W34m no.SL-82-1